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Volume 5 | Technical Appendices

CFA15 | Greatworth to Lower Boddington

River Cherwell at Edgcote modelling report (WR-004-007)

Water resources

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Department for Transport

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1 Introduction

1.1 Structure of the water resources and flood risk assessment appendices

1.1.1 The water resources and flood risk assessment appendices comprise six parts. The first of these is a route-wide appendix (Volume 5: Appendix WR-001-000).

1.1.2 Specific appendices for each community forum area (CFA) are also provided. For the Greatworth to Lower Boddington area (CFA15) these are:

- a water resources assessment (Volume 5: Appendix WR-002-015);
- a flood risk assessment (Volume 5: Appendix WR-003-015); and
- hydraulic modelling reports for the Culworth Brook at Lower Thorpe (Volume 5: Appendix WR-004-006), the River Cherwell at Edgcote (i.e. this appendix), and the Highfurlong Brook (Volume 5: Appendix WR-004-008).

1.1.3 Maps referred to throughout the water resources and flood risk assessment appendices are contained in the Volume 5, Water Resources and Flood Risk Assessment Map Book.

1.2 Scope and structure of this assessment

1.2.1 This document presents an assessment of the fluvial flood risk of the River Cherwell at Edgcote, for the existing (baseline) and post-development (Proposed Scheme) scenario at Trafford Bridge. A two-dimensional model has been developed using hydraulic modelling software TUFLOW in order to investigate the impact of the proposed piers within the floodplain and advise the flood risk assessment.

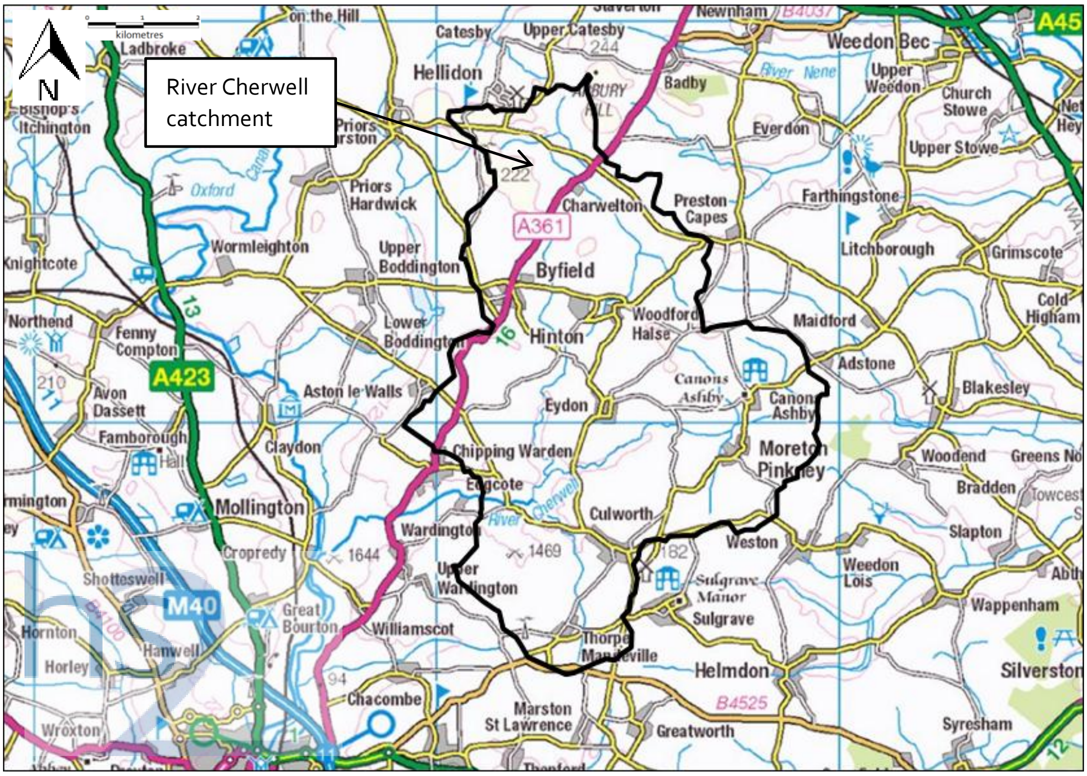
1.2.2 The catchment hydrology is reported in Section 2. Flood water levels, depths and floodplain extents are reported for the baseline (Section 3) and scheme scenarios (Section 4). Section 5 includes conclusions and recommendations and Section 6 covers assumptions and limitations of the hydrology and hydraulic modelling.

2 Hydrology

2.1 Location plan and topography

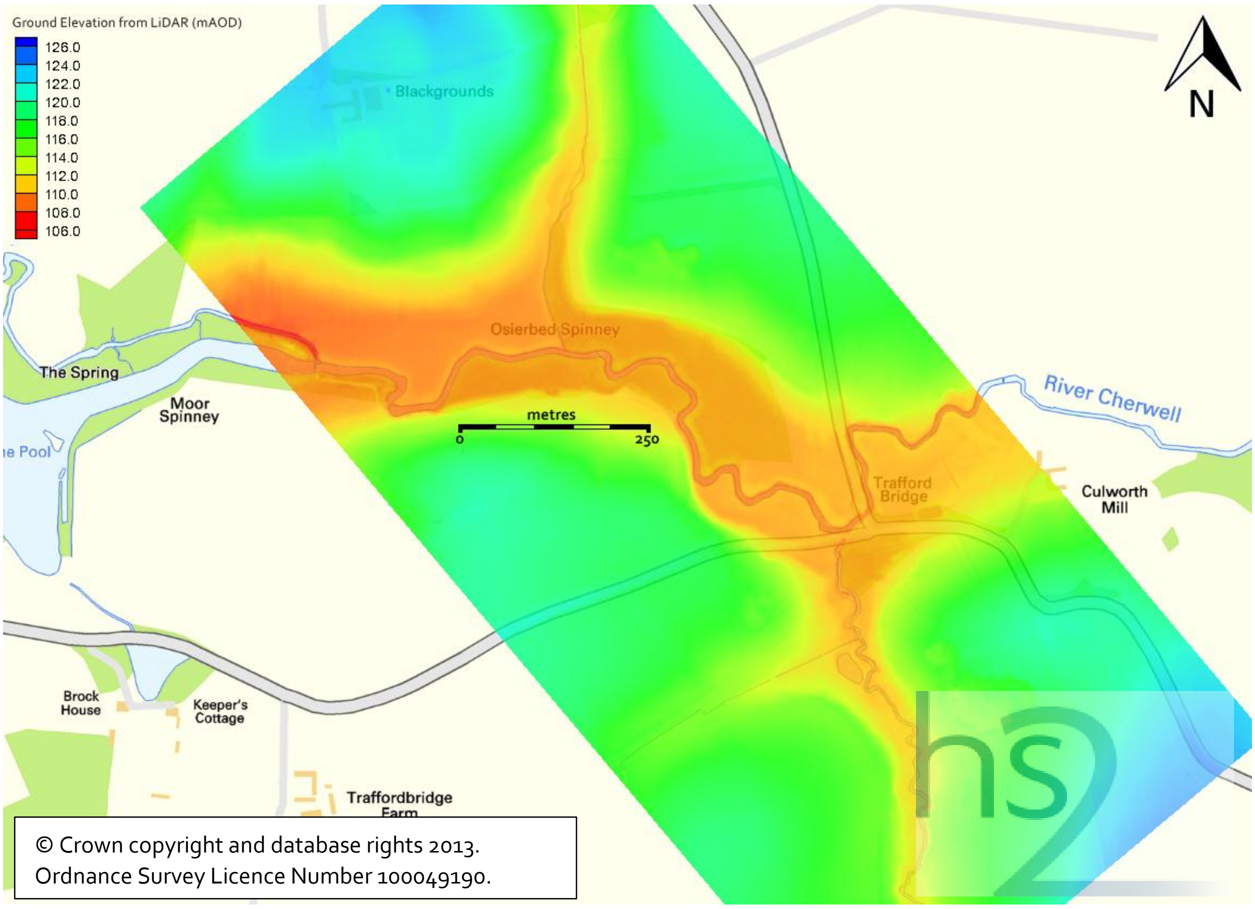
2.1.1 The study catchment is within the rural upper reaches of the River Cherwell upstream of its confluence with the Highfurlong Brook and north-east of Banbury in Northamptonshire, as shown in Figure 1. Upstream of the study area three small branches combine to form the main River Cherwell channel and two further tributaries join within the area of interest. The downstream boundary of the study catchment has been chosen at 'The Pool' upstream of Edgcote village.

Figure 1: Location of the River Cherwell catchment



2.1.2 The catchment of the River Cherwell within the study area contains a number of small villages including Thorpe Mandeville, Culworth, Moreton Pinkney, Hinton and Byfield. The topography of the catchment is relatively flat along the wide river basin with undulating hills in all up-catchment directions. Ground levels within the study area range from approximately 108m above Ordnance Datum (AOD) within the river basin, to in excess of 124m AOD on high ground to the north, east and southern sides. Figure 2 illustrates the light detection and ranging (LiDAR) information, scaled from the 20cm digital terrain model (DTM) to a grid size of 1m for use throughout this modelling exercise.

Figure 2: Topography of the River Cherwell catchment



2.2 Hydrological context

2.2.1 The River Cherwell has a catchment size of 81.7km^2 at 'The Pool' downstream of the crossing location of the Proposed Scheme close to Edgcote. The catchment is relatively shallow (index of catchment steepness (DSPBAR) = 43.4), which is reasonable given its size and location. The longest drainage path is 18.1km, with the average at 8.8km, suggesting some variability in the catchment width along its length.

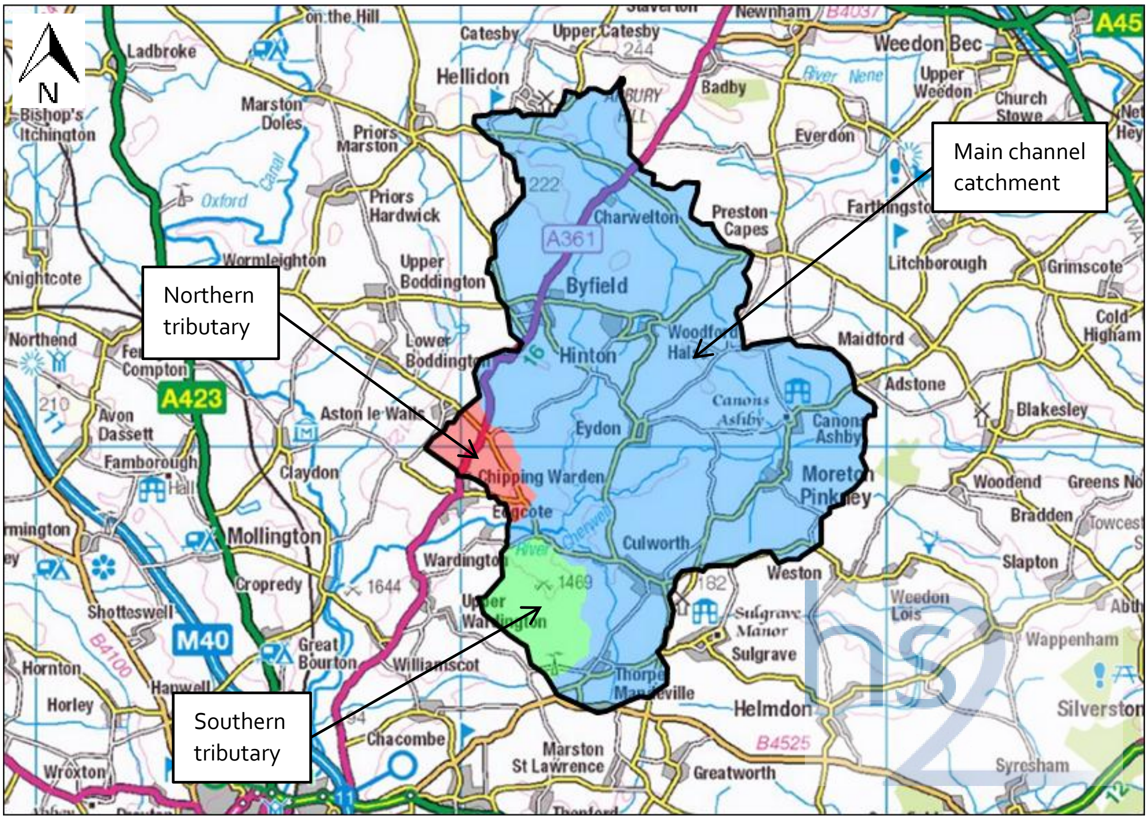
2.2.2 The climate and soils descriptors show that the catchment is relatively dry, with low annual rainfall (standard average annual rainfall (SAAR) = 677mm) and a low proportion of time annually where soils are 'wet' (index of proportion of time soils are wet (PROPWET) = 0.3). There is a small amount of recorded attenuation due to reservoirs or lakes within the catchment (flood attenuation due to reservoirs and lakes (FARL) = 0.984).

2.2.3 The catchment is 'essentially rural' with an urban extent value (in the year 2000) of 0.0112. Urban areas comprise of Thorpe Mandeville and Culworth to the south, Moreton Pinkney and Canons Ashby to the east, and Woodford Halse, Byfield and Charwelton to the northern edge of the catchment.

2.2.4 The natural catchment at the location of interest is composed of the primary watercourse inflow and two smaller inflows from local streams, as shown in Figure 3:

- the main River Cherwell channel at this location comprises three small catchments which join upstream of the study area. These branches originate from Woodford Halse and Byfield to the north, Canons Ashby to the east and Thorpe Mandeville to the south. The combined area of these catchments, which form the main channel for the purposes of this model, is approximately 72.6km^2 (shaded blue in Figure 3);
- a tributary originating within farmland to the south of Edgcote joins the River Cherwell at Trafford Bridge from the southern side. This tributary is approximately 5.8km^2 at its confluence with the Cherwell and is termed the "southern tributary" for the purpose of this report (shaded green in Figure 3); and
- a second tributary joins the Cherwell within the study area from the north. This is a catchment approximately 3.3km^2 in size, and originates north of Chipping Warden. It is termed the "northern tributary" (shaded red in Figure 3).

Figure 3: River Cherwell catchment overview map



2.2.5 The catchments of the lateral inflows have been checked to ensure that the combined area corresponds to the total area at the downstream boundary.

2.3 Hydrological assessment

2.3.1 An initial hydrological assessment was undertaken across the entire River Cherwell catchment to the downstream boundary at 'The Pool' upstream of Edgcote to

determine likely peak flows within the watercourse. A full routed rainfall-runoff output, such as that derived using the Revitalised Flood Hydrograph (ReFH) method, is required for time-varying hydrodynamic modelling. Scaling was used to add an uplift of 20% to the 100 year return period to account for climate change.

ReFH rainfall runoff method

2.3.2 The ReFH rainfall runoff method was applied, using the spreadsheet implementation (v1.4) provided by HR Wallingford, together with the catchment descriptors obtained from the (Flood Estimation Handbook) FEH CD-ROM (v3). No rainfall or flow data were available for the catchment and all ReFH design standard parameters were therefore applied without observed or analogue adjustments. A model timestep of 1 hour and storm duration of 11 hours was used in the analysis of the entire River Cherwell catchment upstream of 'The Pool'.

2.3.3 The FEH depth-duration-frequency rainfall modelling for the catchment was used to obtain total rainfall volumes for each design storm, which was spread across the chosen storm duration using the winter storm profile, due to the rural nature of the catchment. Seasonal correction and areal reduction factors of 0.70 and 0.93 were applied, with resultant total and peak storm rainfall as shown in Table 1.

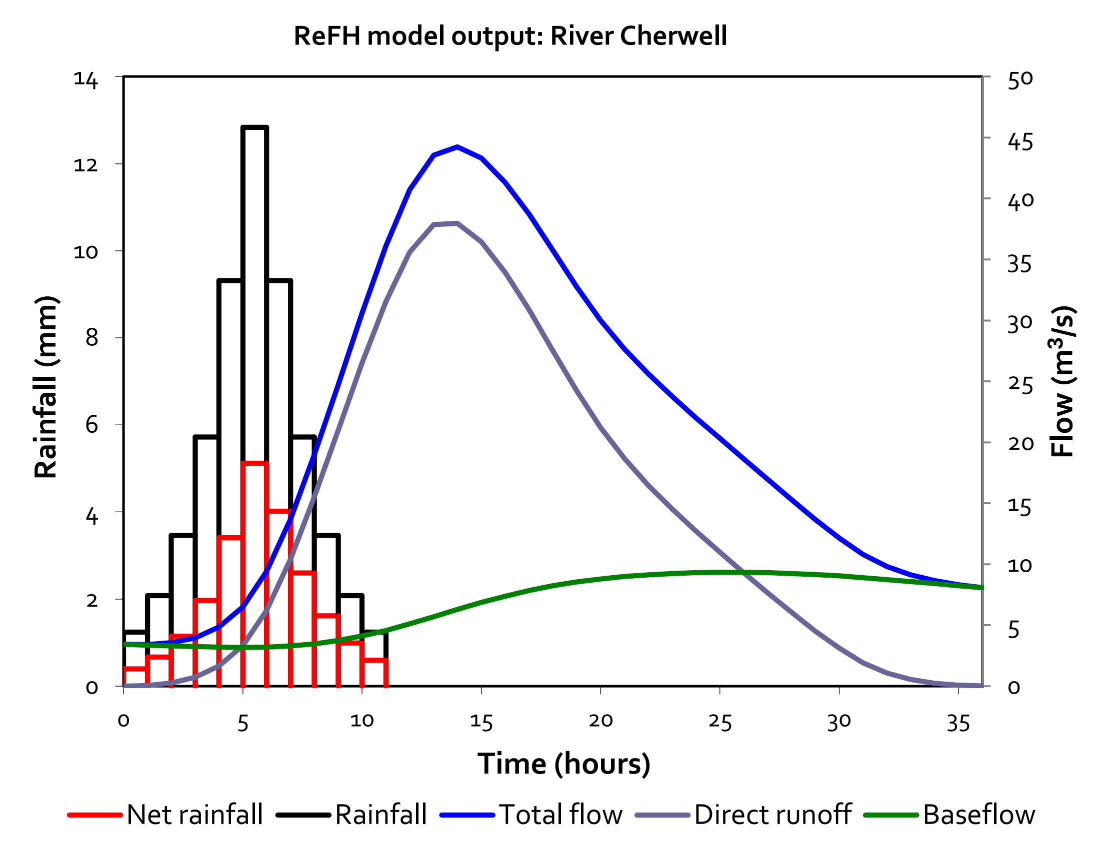
2.3.4 The loss, routing and baseflow models use the catchment descriptors and standard ReFH models and parameters in the absence of any gauged flow information for the watercourse. The unit hydrograph time to peak is 14 hours, which is reasonable for a catchment of this size. Calculated initial baseflows are reasonably low, at approximately 3.4m^3/s. An initial soil moisture deficit of 128mm was calculated. The combined models were applied to the calculated input rainfall by scaling and aggregating the unit hydrograph calculated using the loss and routing models. The baseflow hydrograph was then combined with the storm hydrograph to give a design hydrograph for each return period.

Table 1: River Cherwell (whole catchment) ReFH rainfall volumes and peak flows

Return period	Depth-duration-frequency rainfall (mm)	Design rainfall (mm)	Peak rainfall (mm)	Peak runoff (m^3/s)
20 years	59.3	38.4	8.7	31.8
100 years	87.2	56.5	12.8	44.2
1000 years	150.7	97.6	22.2	76.9

2.3.5 The 100 year return period rainfall event results in a peak runoff rate of 44.2m^3/s for the entire catchment upstream of 'The Pool'. The rainfall hyetograph and corresponding fluvial flood hydrograph are presented in Figure 4.

Figure 4: River Cherwell (whole catchment) ReFH hydrograph for the 100 year return period event



Modelled sub-catchment inflows

2.3.7 There is a main channel and two separate inflows into the River Cherwell within the study area (as described in Section 4.2 of this report). Each sub-catchment inflow must be applied separately at the relevant boundary inflow location in order to accurately model the watercourse and the impact of the Proposed Scheme. Consequently, it is necessary to divide the whole catchment into the component parts to obtain each separate inflow.

2.3.8 The catchment descriptors for the main channel catchment were extracted by subtracting the component sub-catchment parts, for the southern and northern tributaries as identified by the FEH CD-ROM, from the whole catchment. Area-weighted averages, or specific adjustment procedures as detailed in the FEH volume 5[1], were used to generate a catchment descriptor file for the main channel only by removing the tributaries, as presented in Table 2.

[1] Institute of Hydrology (1999), *Flood Estimation Handbook*.

Table 2: Key model catchment descriptors for the River Cherwell and its sub-catchments

Model catchment descriptors	River Cherwell (direct, whole catchment)	Northern catchment (direct)	Southern catchment (direct)	River Cherwell main channel only (derived)
AREA	81.66	3.30	5.75	72.61
BFIHOST	0.41	0.49	0.42	0.40
DPLBAR	8.75	2.23	2.27	10.47
DPSBAR	43.40	40.40	49.40	43.06
FARL	0.98	1.00	1.00	1.00
FPEXT	0.079	0.09	0.07	0.08
FPDBAR	0.66	0.58	0.61	0.67
PROPWET	0.30	0.30	0.30	0.30
SAAR	677	663	673	678
SPRHOST	41.42	37.29	42.42	41.34
URBEXT2000	0.011	0.00	0.00	0.01

2.3.10 The two sub-catchment inflows from the northern and southern tributaries were subtracted from the total catchment in order to derive a 'main channel' catchment. ReFH flows were calculated for each of the three inflows, using individual critical storm durations appropriate to each separate catchment. For the main channel of the River Cherwell, this resulted in critical storm duration of 13 hours, based on a 1 hour timestep, which produced a modelled peak flow at 16 hours. This is a longer critical storm duration than for the 'whole catchment' due to the removal of the two smaller tributaries, which result in a longer average drainage path length of 10.5km. This was imported into the hydraulic model and applied at the upstream boundary.

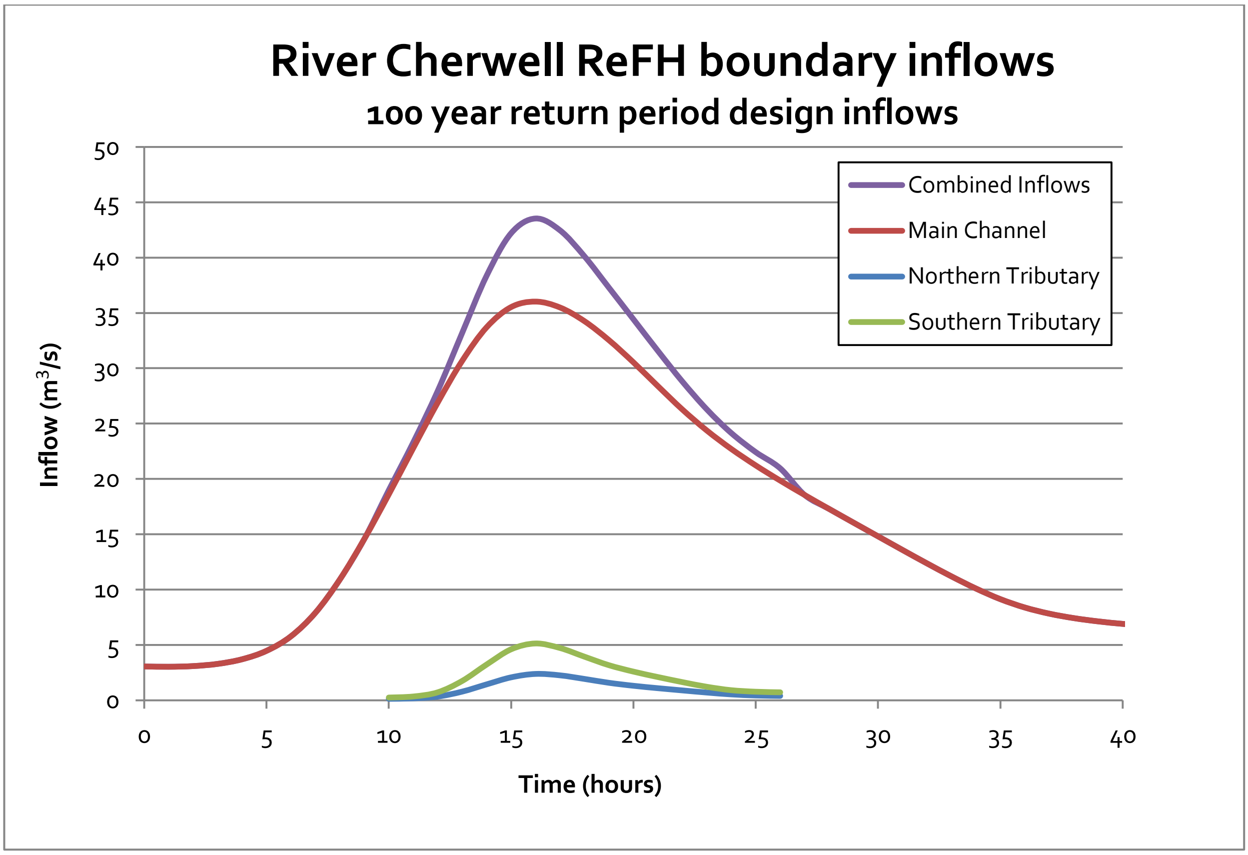
2.3.11 Modelled inflows calculated for each catchment for a number of return periods are presented in Table 3. In order to account for climate change an uplift of 20% was applied to peak 100 year return period flow.

Table 3: River Cherwell modelled ReFH inflows (cumecs)

Return period	ReFH inflows (m^3/s)		
	River Cherwell (main channel)	Northern tributary	Southern tributary
20 years	26.5	1.7	3.6
100 years	36.0	2.4	5.1
100 years plus 20%	43.2	2.9	6.5
1000 years	60.0	4.3	9.3

2.3.12 The two sub-catchments are small relatively steep tributaries with faster response times and consequently, shorter modelled critical storm durations (5.5 hours duration and 30 minute timestep) than the main channel flow through the River Cherwell. The flow hydrographs were imported into the hydraulic model and applied at the northern and southern boundaries. In order to obtain the 'worst-case' scenario at the study area, the peak flow for each tributary was lagged in order to coincide with the peak of the main channel flow and produce a combined maximum flow, as illustrated in Figure 5.

Figure 5: River Cherwell lagged ReFH inflows for the 100 year design rainfall.



2.3.13 The combined peak inflow for the 100 year return period event was found to be 43.5 m^3/s, which corresponds well with the peak flows calculated for the entire catchment. Results of the initial 100 year TUFLOW run confirm that the peak flow into the model is 43.5 m^3/s at 16 hours.

3 Baseline hydraulic modelling

3.1 Model definition

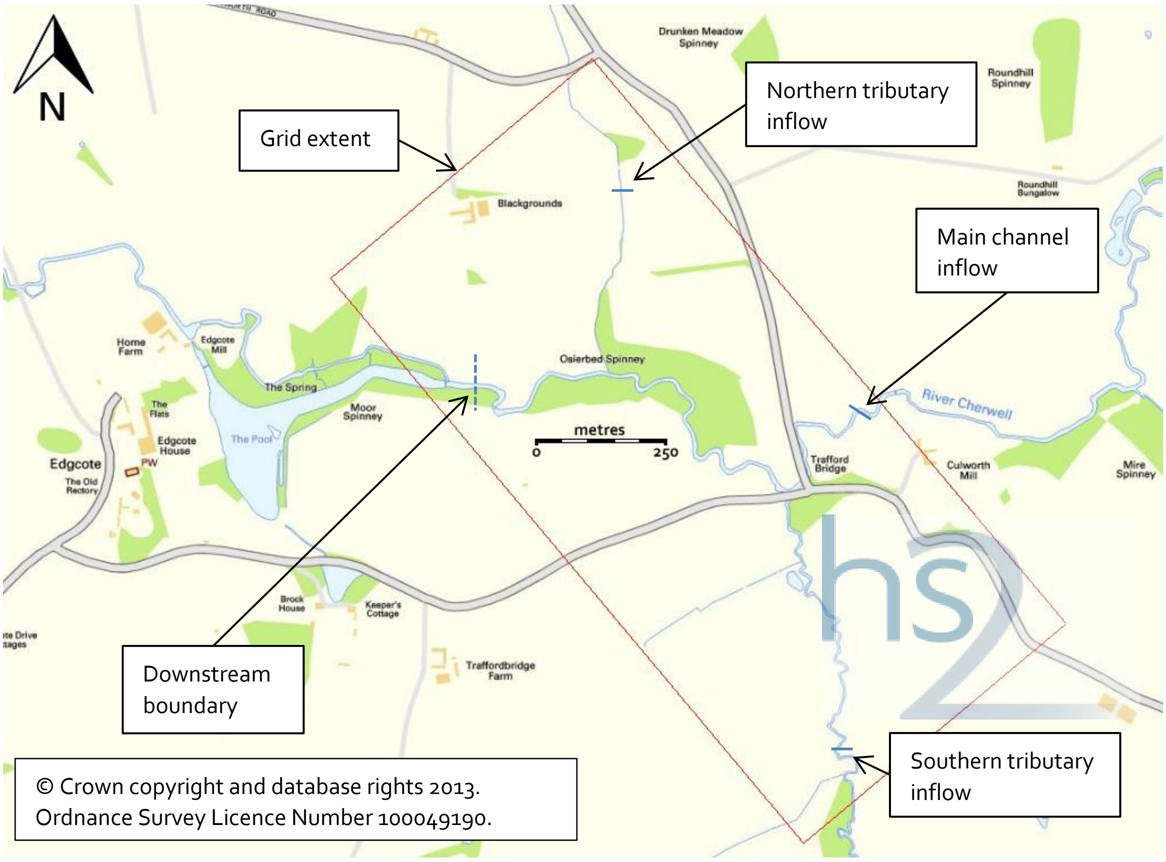
3.1.1 A hydraulic model has been constructed using SMS-TUFLOW software. The model utilised the two-dimensional capabilities of TUFLOW (Version 2012-05-AE, iDP, w64), and the convenience of the Aquaveo SMS (Version 11.0) software for visualisation purposes. The River Cherwell floodplain in this location is wide and flat, with complex overland flow patterns as a result of the tributaries. Since the Proposed Scheme includes piers within the floodplain, it was deemed a two-dimensional model was necessary in order to accurately represent overland flow patterns. Given the nature of the modelling exercise, in-channel flow is not a specific concern and an imbedded one-dimensional channel was not considered necessary.

3.1.2 The topography of the model is based upon 1m resolution LiDAR data which was sampled from the 20cm DTM. A uniform TUFLOW grid size of 3m was used since this was considered appropriate given the size and scale of the model and still gave acceptable run times. No modifications have been made to the cell resolution or base LiDAR.

3.2 Model boundaries

3.2.1 The extent of the TUFLOW model and placement of selected boundaries and inflows, in relation to Edgcote village, is shown in Figure 6.

Figure 6: Overview of the River Cherwell model at Edgcote



3.2.2 The upstream boundary of the main channel of the River Cherwell has been defined as a flow-time boundary. The two tributaries were also added as flow-time boundaries, positioned at the northern and southern edges of the model. Corresponding calculated ReFH hydrographs were applied to each inflow for the 20 year, 100 year including uplift for climate change and 1000 year events.

3.2.3 A flow-head boundary has been applied to the downstream extent of the model and has been automatically generated by TUFLOW based on an assumed gradient of 0.001 within the wide, flat river basin.

3.3 Roughness coefficients and structural definitions

3.3.1 The whole two-dimensional domain has been classified as 'grassland' and assigned a Manning's 'n' value of 0.035. This ensures all floodplain is represented as grassland except where specified otherwise. This was chosen from a normal value for pasture floodplain in accordance Chow (1959)[2]. Since this is an impact assessment, roughness values at the lower end of what is deemed reasonable and appropriate were chosen in order to produce the largest relative impact.

3.3.2 Ordnance Survey (OS) Mastermap layers were imported into SMS in order to define different land uses, and further detail has been added to refine Manning's 'n' value for

[2] Ven Te Chow (1959, 2009 edition). *Open-channel hydraulics*, The Blackburn Press, Caldwell, NJ, USA.

the water surface, roads, trees and buildings. The values used are 0.04, 0.02, 0.1 and 1.0 respectively.

3.3.3 There are two roads present within the modelled area and an arch-bridge which allows Welsh Road to cross the River Cherwell. The LiDAR DTM shows that although both roads are raised on embankment and provide some barrier to flows, these are quickly overtopped by out-of-bank flows from the main channel and the southern tributary. Therefore it was not deemed necessary, given the purpose of this modelling exercise, to represent this in any further detail and structures were not added beneath the highway.

3.4 Baseline model results

Flooding mechanisms

3.4.1 Hydraulic modelling results for the simulated return periods were first viewed within SMS, which allows the user to watch the flood flows as the simulation progresses. Although initially in-channel through the study area, flood water quickly overtops the banks and flows overland across the floodplain. The southern tributary joins the main channel close to the road junction at Trafford Bridge. Despite providing an initial restriction, both roads are quickly overtopped by peak flows at all of the return periods modelled.

3.4.2 Downstream of the road junction and Trafford Bridge, flood water flows in a north-west direction and is contained within the wide river basin. Out-of-bank flood water meets flows from the northern tributary and continues flowing parallel to the channel until it reaches 'The Pool' at the downstream boundary.

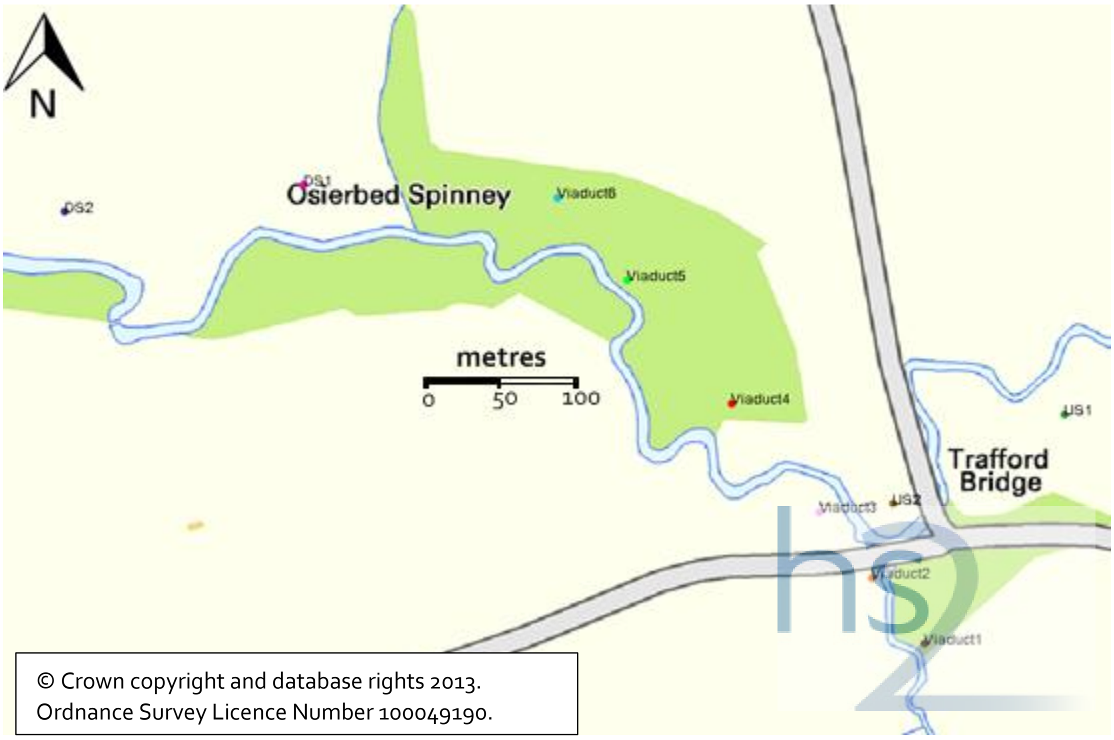
Flood depths and levels

3.4.3 Modelled flood water levels and depths were extracted from chosen points. Additionally, model flows were checked at various locations to ensure that the flow balance was sensible across the grid.

3.4.4 Observation points were selected to represent results at a range of locations and used to extract data at all return periods in both the existing and scheme scenarios.

3.4.5 These include one upstream of Welsh Road, one immediately downstream close to the confluence with the southern tributary, six points running from south-east to north-west beneath the location of the proposed viaduct, one downstream of the confluence with the northern tributary and finally one towards the downstream extent. Observation point locations can be seen in Figure 7.

Figure 7: Observation points location plan



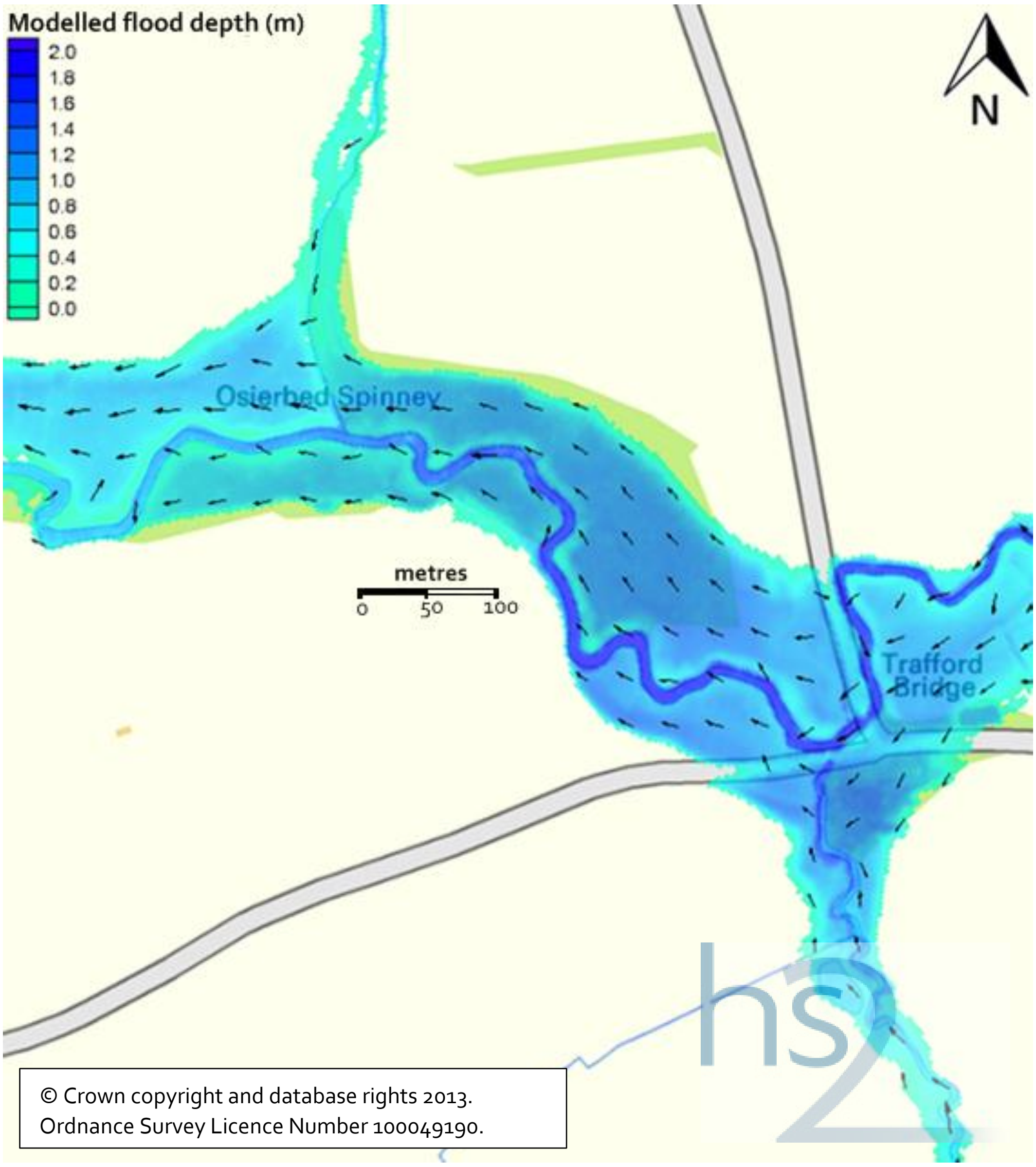
3.4.6 The maximum modelled flood water levels and depths, extracted from the two-dimensional model for all return periods, are recorded in Table 4.

Table 4: Baseline modelled maximum flood water levels and depths

Observation point	Peak 5% annual exceedance probability (AEP)		Peak 1% AEP including climate change		Peak 0.1% AEP	
	Water level (m AOD)	Flood depth (m)	Water level (m AOD)	Flood depth (m)	Water level (m AOD)	Flood depth (m)
Upstream 1	110.04	0.62	110.26	0.83	110.44	1.02
Upstream 2	109.97	0.72	110.21	0.96	110.41	1.16
Viaduct1	109.98	0.71	110.22	0.95	110.42	1.15
Viaduct2	109.98	0.72	110.21	0.96	110.42	1.16
Viaduct3	109.95	0.66	110.20	0.92	110.40	1.12
Viaduct4	109.92	0.99	110.17	1.24	110.37	1.44
Viaduct5	109.80	0.57	110.03	0.80	110.22	0.98
Viaduct6	109.66	0.84	109.88	1.05	110.06	1.23
Downstream 1	109.09	0.61	109.27	0.80	109.42	0.94
Downstream 2	108.68	0.64	108.88	0.85	109.06	1.02

3.4.7 A visual illustration of the flood mechanisms and modelled flood depths for the existing 1 in 100 years return period event (plus uplift of 20% for climate change) can be seen in Figure 8.

Figure 8: 100 year plus climate change existing flood depths (m)



Floodplain extents

3.4.8 Floodplain extents for the 100 year plus climate change and 20 year baseline scenarios are shown on Map WR-05-038 and Map WR-06-038 respectively (Volume 5, Water Resources and Flood Risk Assessment Map Book).

Flood velocities

3.4.9 Modelled flood velocities are generally low within the floodplain and considerably higher within the channel. Velocities are on average approximately 0.4m/s in the floodplain adjacent to the meandering section where flows run north-west. Within the channel, flow velocities are shown to reach in excess of 1m/s.

Sensitivity testing

3.4.10 In order to verify model results and confirm choices made within this modelling exercise, sensitivity analysis was undertaken to test to implications of increasing Manning's roughness in the floodplain.

3.4.11 The floodplain in this area consists of pasture grassland and areas of trees which are shown clearly on the OS mapping. Since access to this land was not permitted at the time of the walkover surveys within this study area, this could not be verified, and as such 'normal' roughness values were chosen based upon Chow (1959). In this publication the 'normal' value for pasture with high grass is 0.035, slightly lower than the project recommended value of 0.05 for two-dimensional modelling of floodplain. This lower value was chosen since this modelling exercise aims to test the impact of piers within the floodplain, and therefore is it important that flow velocities are at the quicker end of what is deemed reasonable and appropriate in order to have the largest possible impact as a result of the pier placement. In addition, based on aerial photography, this appears to be a reasonable estimation for the grass floodplain. Trees are represented in the modelling using a Manning's roughness value of 0.1 which indicates heavy stands of timber with little undergrowth and flood stage below the level of the branches. Areas of trees are identified by OS mapping, and were confirmed based on available aerial photography.

3.4.12 The model was tested by increasing the Manning's roughness values to the upper end of reasonable limits within the floodplain. Grassland was increased to 0.07, which represents the 'maximum' limit for floodplain with scattered brush and heavy weeds. The areas of trees were increased to 0.16, which indicates the upper extent of roughness for trees, assuming the flood water level reaches the branches. Other roughness values (roads, buildings and the water surface) were not altered since these are less important than the floodplain in this scenario, and subject to less variation throughout the year.

3.4.13 The 100 year plus climate change existing scenario was run using the roughened floodplain values. Increasing the roughness caused some instability issues due to the flat nature of the downstream boundary and therefore this was steepened in order to encourage the model to run smoothly. This alteration is not considered to have any impact on the area of interest.

3.4.14 At the location of the route crossing there was an increase in flood water level to 110.46m AOD at observation point 'Viaduct 3', approximately 260mm deeper than the same point for the smoother floodplain. This is to be expected given the significant increase in Manning's. Flood water velocities within the rougher floodplain are 0.2m/s on average in the area, up to 0.2m/s slower than within the modelled smoother floodplain.

3.4.15 The purpose of this modelling exercise was to test the impact of the Proposed Scheme on flood water levels as a result of the placement of piers within the floodplain, not to define existing baseline flood water levels. The impact of the increased roughness can be tested on the Proposed Scheme modelling in order to compare the impact between the existing and proposed results using both rough and smooth floodplains.

3.4.16 In addition to the sensitivity testing of Manning's 'n' within the floodplain, the implications of changing the water surface slope associated with the downstream boundary was tested. Alterations to the downstream boundary were shown to affect levels only immediately upstream of the boundary and these effects did not reach the area of interest. The boundary was therefore deemed sufficiently downstream of the Proposed Scheme.

3.4.17 Visual inspections of the results were carried out for all simulated return period to ensure that the results looked to be appropriate, and visual comparisons were made to Environment Agency Flood Zones for the River Cherwell.

4 Scheme scenario

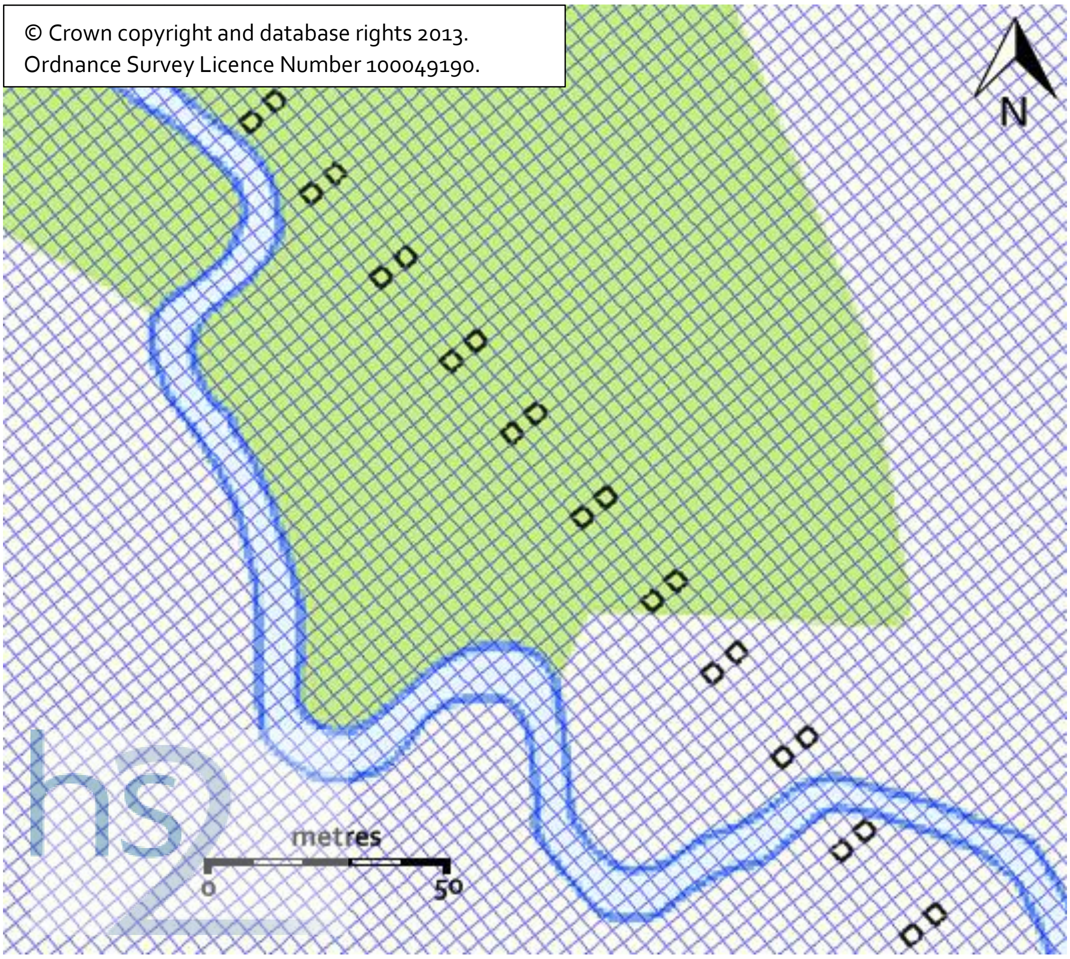
4.1 Scheme modelling methodology

Hydraulic modelling

4.1.1 The purpose of this modelling exercise is to assess the impact of the Proposed Scheme viaduct piers on flood water levels in the vicinity of Trafford Bridge.

4.1.2 The piers of the Proposed Scheme viaduct across the River Cherwell have been added to the model.

Figure 9: Piers modelled as inactive TUFLOW grid cells



4.1.3 There are no structures present within the scheme scenario TUFLOW model, and there have been no other changes from the baseline model.

4.2 Proposed Scheme model results

Flood depths and levels

4.2.1 Peak levels and depths have been extracted from the same observation points as before. Changes in afflux as a result of the Proposed Scheme scenario have been included in Table 5.

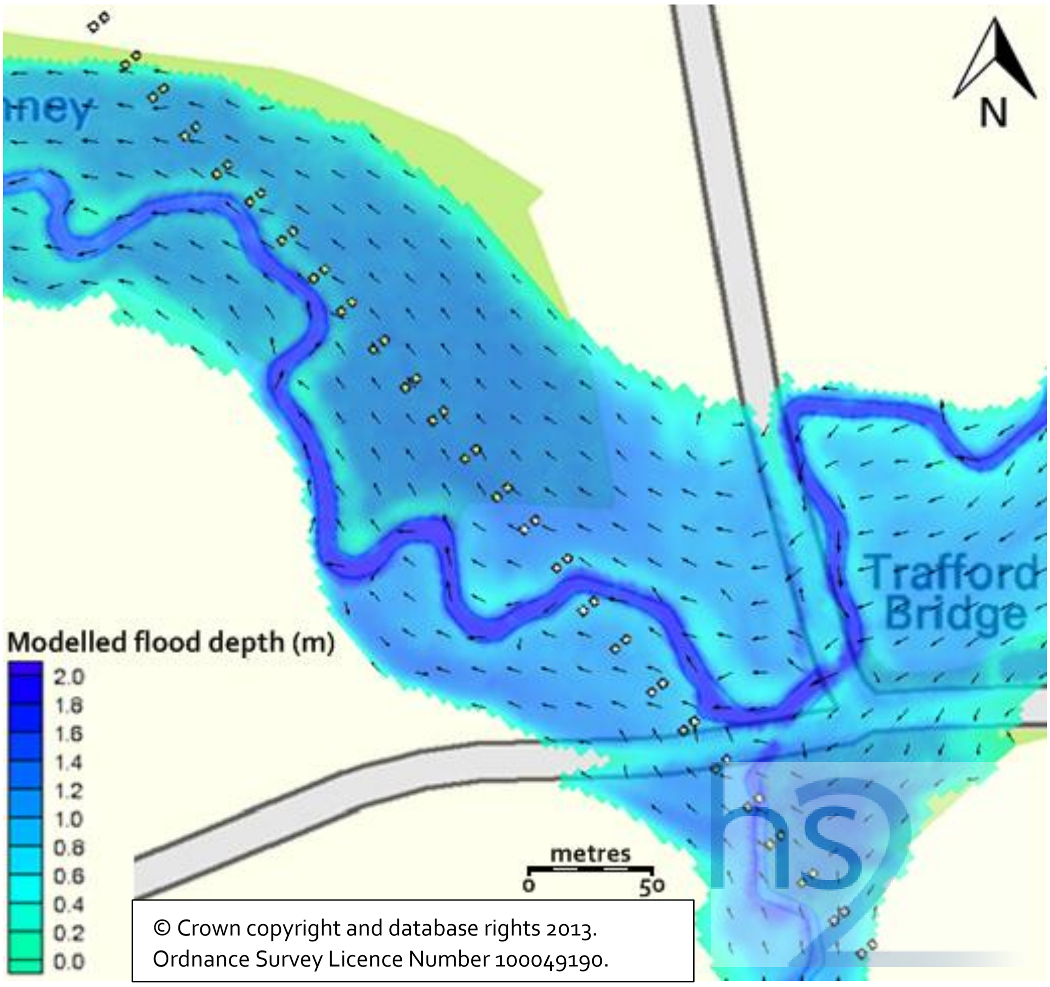
Table 5: Scheme modelled maximum flood levels, depths and increases in afflux

Observation point	Peak 5% AEP			Peak 1% AEP plus climate change			Peak 0.1% AEP		
	Water level (m AOD)	Flood depth (m)	Afflux (m)	Water level (m AOD)	Flood depth (m)	Afflux (m)	Water level (m AOD)	Flood depth (m)	Afflux (m)
Upstream 1	110.05	0.62	0.006	110.27	0.84	0.012	110.46	1.03	0.015
Upstream 2	109.98	0.73	0.010	110.23	0.98	0.015	110.43	1.18	0.017
Viaduct1	109.99	0.72	0.011	110.23	0.96	0.015	110.44	1.17	0.018
Viaduct2	109.99	0.73	0.011	110.23	0.97	0.015	110.43	1.18	0.018
Viaduct3	109.96	0.68	0.013	110.21	0.93	0.016	110.42	1.14	0.018
Viaduct4	109.93	1.00	0.010	110.18	1.25	0.013	110.39	1.46	0.015
Viaduct5	109.81	0.57	0.009	110.05	0.81	0.011	110.24	1.00	0.013
Viaduct6	109.66	0.84	0.002	109.88	1.06	0.001	110.06	1.24	0.002
Downstream 1	109.09	0.61	0.000	109.27	0.80	0.000	109.42	0.94	0.000
Downstream 2	108.68	0.64	0.000	108.88	0.85	0.000	109.06	1.02	0.000

4.2.2 The results indicate that there is a minor increase in flood water level upstream of the Proposed Scheme as a result of the placement of piers within the watercourse.

4.2.3 A visual illustration of the flood mechanisms, flow directions and modelled flood depths as a result of the Proposed Scheme piers in the 1 in 100 years return period event (plus uplift of 20% for climate change) can be seen in Figure 10.

Figure 10: 100 year plus climate change scheme scenario flood depths (m)



Flood extents

4.2.5 Outlines of floodplain extents for the 100 year plus climate change and 20 year Proposed Scheme scenarios are shown on Map WR-05-038 and Map WR-06-038 respectively (Volume 5, Water Resources and Flood Risk Assessment Map Book).

4.2.6 Modelled outlines of the Proposed Scheme scenario show negligible change in extent in comparison with the baseline scenario.

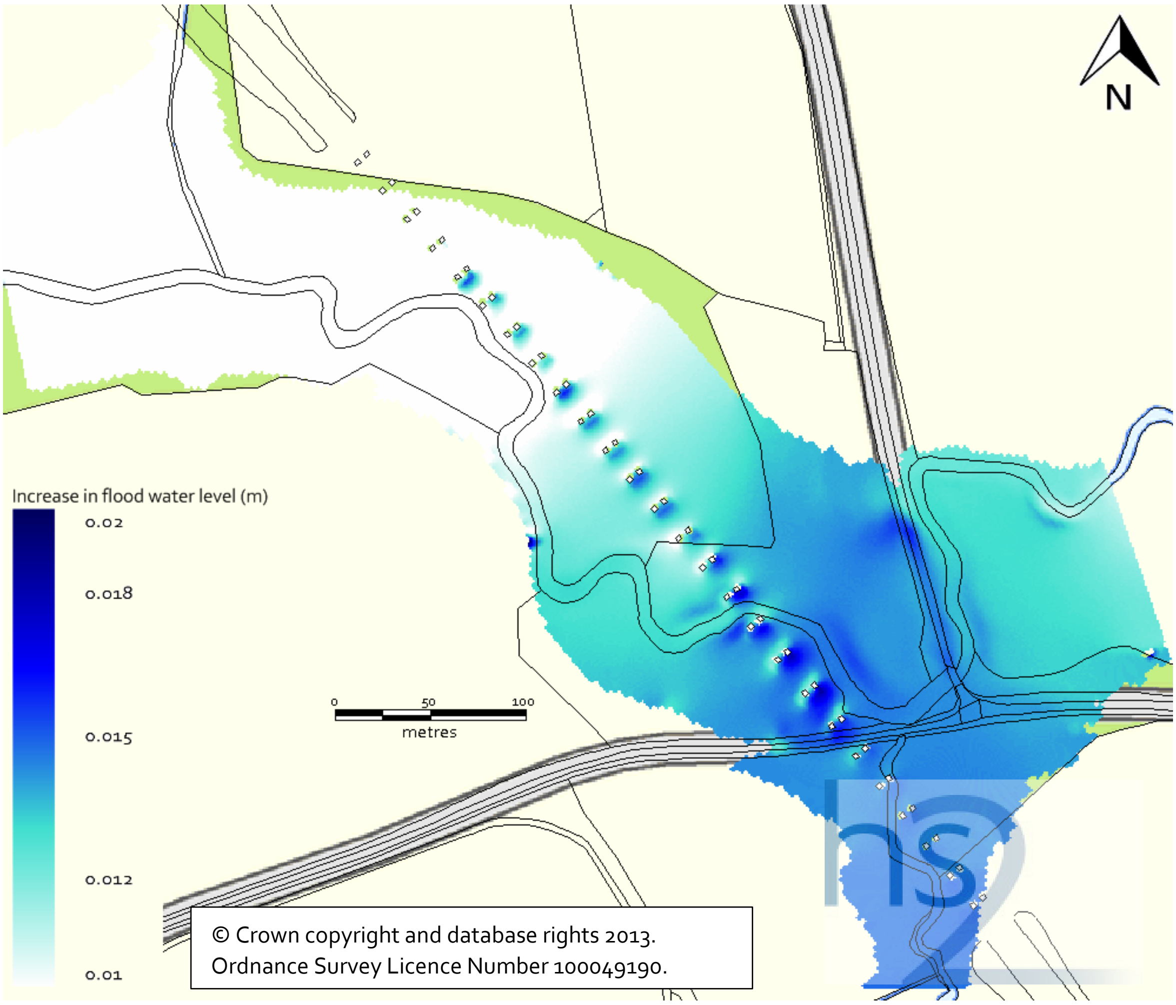
Increase in afflux

4.2.7 Although the increase in flood water level as a result of the Proposed Scheme is minor, further work was undertaken to determine the extent of area impacted. The baseline water levels have been subtracted from the Proposed Scheme levels in order to obtain a grid of the afflux. Increases in excess of 10mm were plotted.

4.2.8 Figure 11 illustrates the increases in afflux as a result of the piers in the 1 in 100 years (1% annual probability) plus climate change event. Increases can be seen upstream of the majority of the piers, and overall there was found to be a maximum increase of approximately 20mm in the depth of flooding. The location of the largest increase was upstream of piers close to the watercourse just downstream of the confluence within the southern tributary.

4.2.9 The flood flows in this area are most complex and are multi-directional as a result of the two inflows and overtopping of the road embankments. This can be seen in Figure 10. It is not surprising, therefore, that the largest impact can be seen in this area. In addition, this area is grassland floodplain. Further downstream, out-of-bank flows run parallel to the orientation of the viaduct and the floodplain is rougher, due to areas of trees. The impact of the Proposed Scheme is less as a result.

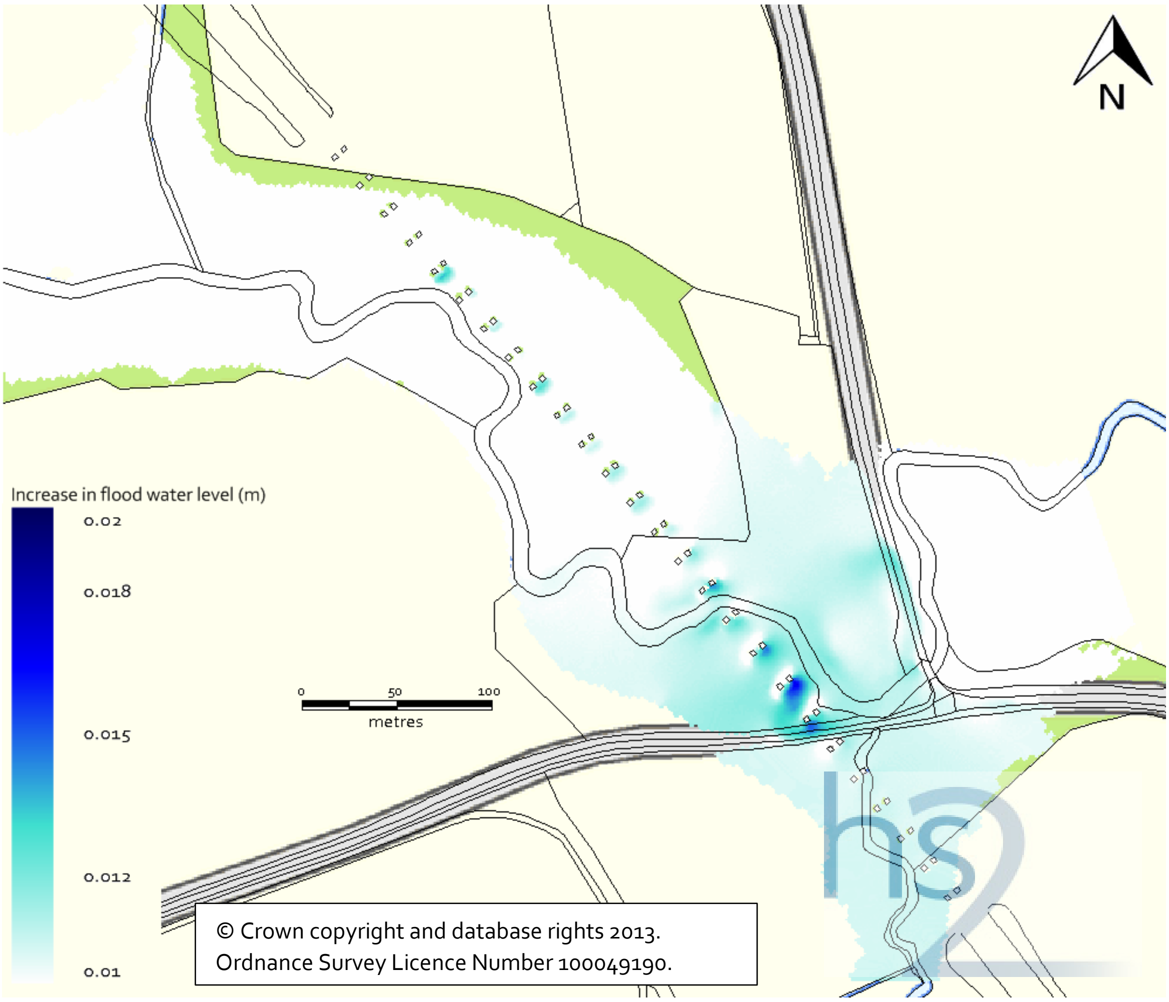
Figure 11: Increase in floodwater level in 100 year plus climate change flood event



4.2.10 The increase in afflux can be seen upstream of both road embankment in the 100year plus climate change event, in the main channel and the southern tributary. In lower return periods the extent of this impact is less and Figure 12 illustrates that this impact does not reach upstream of Welsh Road.

4.2.11 In the 20 year event generally, the increase in afflux is less than in larger events, as would be expected. The small increases are focused immediately upstream of the piers and in the area around the confluence with the southern tributary. Increases in the wider floodplain are approximately 11mm, which is small in relation to baseline flood depths of 0.7m.

Figure 12: Increase in floodwater level in 20 year flood event



Flood velocities

4.2.12 As a result of the small increase in water level in the vicinity of the viaduct structure, flood water velocities decrease by approximately 0.1m/s in the floodplain.

Sensitivity testing

4.2.13 There are multiple methods of modelling piers within a floodplain in TUFLOW. Two methods were tested during the modelling process; blocking grid cells (inactive cells) and raising cell elevations. Results comparison indicated that there was no difference in modelled flood water levels, therefore the method of blocking cells was chosen for simplicity.

4.2.14 As detailed above, the impact of increasing the roughness of the floodplain was tested by increasing Manning's 'n' values to 0.07 and 0.16 for grassland and tress respectively. Modelling results show flood water levels of 110.47m AOD, with an increase of 14mm when compared to the existing scenario as a result of the piers within the rough floodplain. This is approximately 2mm less than the impact seen as a result of piers within the smoother floodplain.

5 Conclusions

5.1.1 This modelling work was undertaken in order to quantify the impact of placement of piers within the floodplain as a result of the Edgcote viaduct. No existing hydraulic model of the River Cherwell at the study area was available for use, therefore a two-dimensional model has been constructed using SMS-TUFLOW.

5.1.2 Three main flood events have been modelled: the 20 year return period event; the 100 year return event with an allowance of 20% for climate change and the 1,000 year return period flood event.

5.1.3 The baseline modelling illustrates the flood mechanisms from the River Cherwell and the northern and southern tributaries which join the main channel within the study area. Modelling of the Proposed Scheme scenario with piers within the watercourse shows a localised increase in flood water level as a result of the minor obstruction formed by the proposed viaduct piers. A maximum increase of approximately 20mm is shown, with impact localised around the confluence of the main River Cherwell channel and the southern tributary. There is a negligible change in floodplain extent as a result of the increase in afflux.

5.1.4 The model is based solely on the 20cm LiDAR data, which was sampled to a 1m interval for management of the data, and a TUFLOW grid size of 3m was used throughout. The accuracy of the model is considered sufficient to provide the information required for this exercise. The model results should not be used for any purpose other than those specified in this report.

5.1.5 The purpose of this modelling exercise was to quantify the impact of the Proposed Scheme on out-of-bank water levels within the River Cherwell floodplain. It is therefore considered that the detail of modelling undertaken is deemed robust and appropriate and it is not envisaged that further, more detailed modelling will be required for this purpose.

6 Assumptions and limitations

Hydrology

6.1.1 Flow estimation follows the guidance within the FEH in conjunction with the latest guidance on its use provided by the Environment Agency.

6.1.2 Catchment descriptors have been extracted from the FEH CD ROM (v3) and sensibility checks have been undertaken. Where catchments have been derived by subtracting catchments, the area- weighted average method has been used following guidance from the FEH volume 5.

6.1.3 ReFH flows have been calculated throughout and validation or calibration of the calculated flows with gauged records has not been carried out. This modelling exercise aims to test the impact of the Proposed Scheme, rather than define baseline flood water levels.

Use of existing models

6.1.4 No existing modelling was been used.

Hydraulic modelling

6.1.5 Only the assessment of flood risk from the River Cherwell has been presented in this report.

6.1.6 Proposed piers have been represented by blocking flow using inactive grid cells. This is a conservative approach since this representation prevents flow entering the whole 3m by 3m grid cell. This is larger than the proposed pier footprint ($2.5m^2$) and therefore will marginally overestimate the impact.

Topography

6.1.7 The model cell size of 3m differs from that of the resolution of the LiDAR DTM, and therefore the actual model topography is therefore not of as high a resolution as the LiDAR DTM.

6.1.8 No channel is cut within the ground model, and therefore the in-channel level represented is actually the water level picked up by the flown LiDAR. No modifications have been made to the ground model to reduce this level to account for in-channel capacity.

Model parameters

6.1.9 Infiltration losses have not been applied.

6.1.10 Roughness of the study area has been defined using broad Manning's 'n' values for a selection of land use types. These were obtained from OS Mastermap data and confirmed with aerial photography.

6.1.11 The upstream and downstream model extents have been located a sufficient distance from the crossing by the Proposed Scheme to provide sufficient length for stability.

6.1.12 Hydrological inflows have generally been applied in the model as point inflows.

Structures

6.1.13 Due to land access restrictions, no site familiarisation visit or walkover survey has been undertaken for this study area.

6.1.14 Detailed survey of structures beneath the roads present within the study area were not obtained therefore no structures were included within the model.

6.1.15 Modelling of this viaduct crossing has focuses only on the impact of piers within the floodplain. The flood risk assessment (Volume 5: Appendix WR-003-015) concludes that the soffit level of the viaduct is more than 6m above the modelled flood water level. The embankment is located out of the main channel floodplain, and therefore does not form a restriction to flood flows and has therefore not been included. It does however cut off flow from the northern tributary, which is conveyed beneath the embankment in a culvert. Any alteration to flow characteristics as a result is unlikely to have an impact on the main floodplain, and therefore for the purposes of this modelling exercise this culvert has not been included.

Post processing of results

6.1.16 All two-dimensional model results have been processed to a grid resolution of 1m from a model cell size of 3m. This is slightly smaller than the usual resolution of half the modelled cell size. This was done to ensure that the shape of the small piers was not distorted or lost in the post processing.

6.1.17 The TUFLOW flood outlines presented in this report have not undergone any post-processing, such as smoothing of edges or filling in of dry islands.

Validation

6.1.18 Limited sensitivity testing of floodplain roughness has been carried out to confirm values used within the project. Further, full sensitivity analysis an all roughness parameters and boundaries will be undertaken as part of detailed modelling.

6.1.19 Sensibility checks of general flood mechanisms, including flow over the roads within the study area, has been undertaken based on available LiDAR information and aerial photography.

7 References

Institute of Hydrology (1999), *Flood Estimation Handbook.*

Ven Te Chow (1959, 2009 edition). *Open-channel hydraulics*, The Blackburn Press, Caldwell, NJ, USA.